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DESIGN OF A WAREHOUSE IN REINFORCED CONCRETE

BY

HARRY F. ROBINSON

THESIS

For the Degree of Bachelor of Science
in Architecture

COLLEGE OF ENGINEERING
UNIVERSITY OF ILLINOIS

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Harry Franklin Robinson

ENTITLED Design of a Warehouse in Reinforced Concrete

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE

OF Bachelor of Science in Architecture

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HEAD OF DEPARTMENT OF Architecture

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General Remarks.

In designing in reinforced concrete, many things tend to complicate matters. Concrete resists compression strongly, but, as experiments show, it is practically useless in tension. Steel, on the contrary, is very strong in tension, so that it is employed to resist the tension in the beam or floor slab; and the concrete to resist the compression. Results of Experiments have not been uniform, although sometimes very similar ones have been obtained. The proper mixing of concrete is a most important point, wherein differences may occur. The sizes of the aggregates comprising it, the kind of sand, the proportion of the cement sand, and aggregate are important.

In general, it may be said, that for foundation walls or where the concrete does not have to be tamped in closely about steel bars, spaced near to each other, the broken stone or coarse gravel may be of rather large size, and the proportions may be 1-2-6. Where small steel rods are placed close together in beams, the utmost care must be

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later to enclose all the steel bars with Concrete, and the aggregate must be of small pieces, about 1/2" or 3/8" and the proportions should be 1-2-4, or 1-2½-5. Under concrete should not be used for important work, as its properties are too uncertain. Hence, in designing Concrete structures, the architect must specify accurately, the kind of Concrete to be employed, in exact proportions, and size of aggregates and he must see by careful superintendence, that the specifications are obeyed.

There is, in Germany, a Union composed of architects, Contractors and engineers, which meets annually and draws up rules for the design and construction of buildings and it has made some admirable improvements in Concrete specifications. Rules are made concerning the plans and details employed or required; structural drawings by both architect and contractor; the kind of superintendent required; the tests to be made; and practically all points are covered which can possibly be in question.

Nothing like that Union is found in

this country, but each Contractor employs his own system of calculation and construction, and there is not much uniformity in methods. From all opinions expressed on the subject and from recent experiences of Concrete Contractors and Architects, it appears that the only method for the architect in drawing up his specifications for this work, is to make them very definite and consistent. That is: he should state exactly the mixture to be used; brand and grade of cement, size of aggregate and standard of sand. He should name the maximum safe limit of live load for the floor and, most important of all, should state the limit of the unit Compression, the concrete is designed safely to resist. After this is done, each Contractor, estimating on the work, is restricted to these limits but may design and build in accordance with his system, while his design and work remain within the limits specified. Instead of several entirely different hypothetic bases for the design there is but one, and it is much safer and simpler than

to permit each Contractor to use his own discretion.

The responsibility rests on the architect, but that is proper, if the architect must know when the construction is safe.

The architect and contractor should work together and not adversely. In this particular, certainly, the German methods are immeasurably superior to our own. The architects and contractors are there on good terms, work together and the architect is responsible for the safety of the building, while the contractor is merely the employe.

Principles of Design.

Many experiments and calculations have been made on concrete floor slabs and beams, and their results have generally been nearly uniform. The conclusions of three investigators are noticeably similar;- those of Professor A. N. Talbot, Professor F. E. Turneaure, and of Sanford E. Thompson C.E.

These investigators agree in the follow-

ving points: —

1. That tensile resistance of concrete should not be considered.

2. That a plane cross section in a concrete beam before bending, is a plane section after bending.

3. Initial tension or compression is absent in the steel.

4. Adhesion of concrete to steel is perfect.

5. The modulus of elasticity of concrete is constant within the usual limits of stress.

In Taylor and Thompsons work on "Concrete, Plain and reinforced" the following reasons are stated for the adoption of the principles just stated: —

a. Beams designed in accordance therewith and properly built, will unquestionably be safe.

b. Fine cracks are formed in the portion of the beam in tension under small loads and these actually destroy the tensile resistance of the concrete.

c. The modulus of elasticity found in many tests has an approximately con-

slant value up to nearly the breaking point; therefore, until the true curve of values has been determined, it may be assumed to be constant.

d. This theory is the simplest and most easily understood of any before presented.

e. It has been adopted by M. Christophe; by the Russian Dept. of Public works in 1904 and by other authorities.

f. The results obtained by it may be readily compared with those of other theories.

The following formulae are taken from Taylor and Thompson:- In designing beams and floor slabs, at any vertical section through the beam, the concrete in the upper portion resists the forces tending to compress it, and the steel bars in the lower portion resist the forces tending to break them in tension. The compression resistance acts horizontally in one direction, and the tensile resistance in the opposite, as indicated by arrows in the diagram. The center of tension in the steel is at the center or axis of the

steel rods. The center of pressure of the concrete is located at the center of gravity of the triangle representing the compressive stresses.

The notation used is the following:

h = the height of beam in inches.

b = " breadth " " " "

p = " ratio of total cross section area of steel rods to cross section of concrete beam above axis of steel bars.

C = unit of compression in lbs. per sq. inch at uppermost fiber of the concrete.

S = unit tension in lbs. per sq. in. in steel rods.

E_c = Modulus of elasticity of concrete in compression.

E_s = Modulus of elasticity of steel in tension.

$r = \frac{E_s}{E_c}$ = ratio of moduli of elasticity of steel and concrete.

d = distance in inches from the top of beam to the axis of the steel rods.

xd = distance from uppermost fiber in compression to neutral axis of beam with steel rods at a depth d below it.

x = ratio of depth of neutral axis to depth to axis of steel rods.

M_R = Moment of resistance of beam in inch lbs.

M_B bending moment in inch lbs.

k constant for a certain steel and a certain concrete.

c = depth of concrete in inches below the axis of steel bars.

F = factor of safety.

The ordinary American system of weights and measures is employed.

Linear dimensions are all taken in inches.

Surface dimensions are in square inches.

Stress, either compressile or tensile in pounds per square inch.

Deformations are in inches.

Moduli of elasticity are in pounds per square inch.

Moments are in inch pounds.

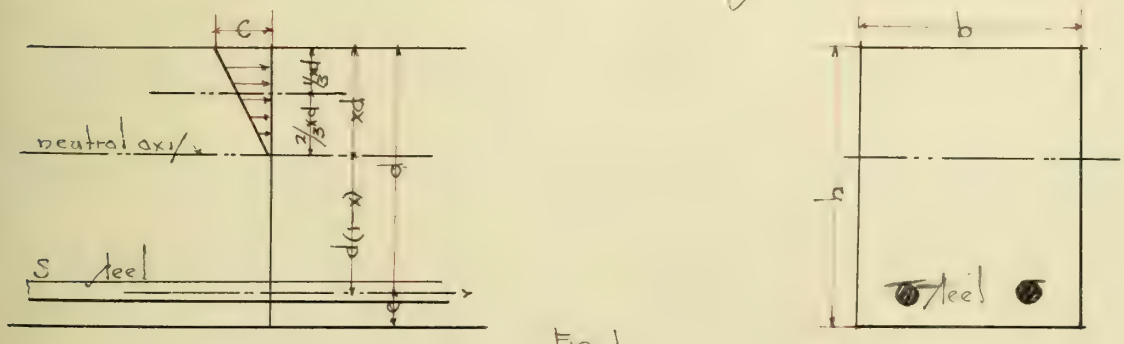

Fig. 1.

Referring to the figure, the internal resisting forces form a couple. The arm repre-

sents the total compression in the concrete, acting at the center of gravity of the stress triangle; the other arm represents the total tension in the steel, acting at the center plane of the rods. For equilibrium these two forces are parallel to each other, have opposed senses and must be equal. In other words, the compression in the concrete equals the tension in the steel. As soon as either one exceeds the ultimate resistance of the material the beam fails.

The moment of resistance of the couple must equal or be greater than the bending moment of the sum of the live and dead loads. Since it is ~~oppos~~ assumed that a plane section before bending remains a plane section after bending, we have the proportion: —

$$\frac{\text{Deformation in steel}}{\text{Deformation in extreme fiber of compression}} = \frac{d(1-x)}{xd}$$

And since deformation = $\frac{\text{stress per square inch}}{\text{modulus of elasticity}}$, we have: —

$$\frac{\frac{S}{E_s}}{\frac{C}{E_c}} = \frac{d(1-x)}{xd} \quad \text{or} \quad \frac{S}{Cr} = \frac{1-x}{x} \qquad (1)$$

From which: — $$x = \frac{1}{1+\frac{S}{Cr}} \qquad (2)$$

Solving formula ① for C, we obtain:

$$C = S\frac{x}{r(1-x)}$$

The total tension in the steel is unit tension S multiplied by the area of the steel pdb; the total compression of the concrete equals the area of the pressure triangle $\frac{1}{2}(C(xd))$ times the breadth of the beam, b. Equating these two stresses and cancelling db in both, we find:—

$$pS = \frac{Cx}{2} \qquad ④$$

Substituting x in ② for x in ④ we have:—

$$p = \frac{1}{2\left(\frac{S}{C}\right)\left(1+\frac{S}{Cr}\right)} \qquad ⑤$$

Hence, for any given percentage of steel area to concrete area, the values of S and C cannot be assumed independently as they bear a constant ratio to each other. Substituting the value of C from formula ③ for C in formula ④ we have:—

$$p = \frac{x}{2}\frac{x}{(1-x)r}$$

Solving this for x we have:—

$$x = -rp + \sqrt{2rp+(rp)^2} \qquad ⑥$$

Thus having x in terms of r and p, by formula ⑥, the location of the neutral axis may

be calculated for any percentage of steel area to concrete area, the steel and concrete having known Moduli of Elasticity.

The moment of resistance is derived from the couple by taking moments about the center of compression in the concrete, using for the force, the total tension in the steel, which as above is $p\,S\,b\,d$, times the arm (see fig 1) $d - x\frac{d}{3}$.

or $$M_R = p\,S\,b\,d^2\left(1 - x/3\right), \qquad (7)$$

The moment may also be deduced by taking moments about the center of tension in the steel.

Thus: — $$M_R = \frac{C\,x\,b\,d^2}{2}\left(1 - x/3\right) \qquad (8)$$

The value of M_R may be expressed without using x by substituting in (7) and (8) the values of (p) from formula (5) and the value of x from formula (2), giving: —

$$M_R = S\,b\,d^2\left[\frac{1}{\frac{2S}{C}\left(1+\frac{S}{Cr}\right)}\right]\left[1 - \frac{1}{3\left(1+\frac{S}{Cr}\right)}\right] \qquad (9)$$

or $$M_R = \frac{C\,b\,d^2}{2}\left[\frac{1}{1+\frac{S}{Cr}}\right]\left[1 - \frac{1}{3\left(1+\frac{S}{Cr}\right)}\right] \qquad (10)$$

In applying formula (10) S and C being corresponding values, cannot be assumed independently of each other, since, for any given percentage of steel to concrete, the ratio of S to

C is constant. For concrete of a given quality if we assume C as representing the unit crushing strength of the concrete, or rather, the compression which we permit the concrete to have, at the time the steel reaches its elastic limit – C is a constant, and E_c is also constant. Likewise for any steel, S and E_s are constant. Consequently r, the ratio of E_s to E_c is constant, and if F equal the required factor of safety, we may place:-

$$k = \frac{S}{F}\left[\frac{1}{\frac{2S}{C}\left(1+\frac{S}{Cr}\right)}\right]\left[1-\frac{1}{3\left(1+\frac{S}{Cr}\right)}\right] \qquad (11)$$

or, in a slightly different form: —

$$k = \frac{C}{2F}\left[\frac{1}{1+\frac{S}{Cr}}\right]\left[1-\frac{1}{3\left(1+\frac{S}{Cr}\right)}\right] \qquad (12)$$

We may thus write, instead of formula (10) the formula (13)

$$M_R = k b d^2 \qquad (13)$$

Since, in any beam, the moment of resistance must equal or excel the bending moment, we may substitute M_B for M_R, also since in the figure 1, $d = h - e$, the formula may be written

$$h = \sqrt{\frac{M_B}{Kb}} + e \qquad (14)$$

Formula (13) is that employed here in the design of floor slabs and girders. Ordi-

many mild steel has been used throughout, for, while high carbon steel would have a higher elastic limit, the modulus of elasticity would be the same, and, in using high carbon steel, tests of the steel should be frequently made. In the table on page 302 in Taylor and Thompson's work - the ratio $\frac{E_s}{E_c}$ is given, the working strength of the concrete, with the safe working values of k for each.

In my thesis design, I have taken 2,000,000. as $\frac{M}{c}$, r = 15, 550 safe resistance of the concrete, giving a value of 83 for k.

Employing these values throughout, and having found the bending moment in each case, it is placed equal to kbd^2; b is assumed, and a value for d is found. Then the ratio of reinforcement becomes .0073; so that multiplying the area ~~by~~ of the concrete by .0073 gives the area of the steel to be used.

In designing my columns, I used the formula as given on page 329.

C_1 = total unit compression upon concrete and steel (the total load divided by the combined area of concrete and steel) in pounds per sq. in.

C - the unit compression on the concrete in pounds per square inch.

p - the ratio of the cross section of the steel to the total cross section of the column.

$r = \frac{E_s}{E_c}$ - the ratio of the moduli of elasticity of the steel and concrete.

Then:

$$C_1 = C[(1-p) + rp]$$

$$p = \frac{C_1 - C}{C(r-1)}$$

The unit compression C was assumed, allowing 500# on the concrete, making $r = 15$, and from this p was determined, which was used in all the columns.

For internal walls (except small vault walls and for silver vault) I used a 4" wall built up of channels (2½") with wire netting stretched over the outside, and the concrete filled in so as to extend 3/4" on the outside of the channels. (See Fig 2.)

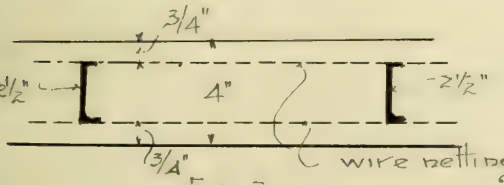

Fig. 2.

For vault partition walls on the second floor and above, used the same construction, putting 1" channels instead of 2½". Two walls 4" thick, were built for

silver vault with a 1" air space between them. The interior was then lined with four layers of 1/8" nickel steel, each layer breaking joint over the one beneath, and curving around the angles of the compartment. (Fig 3)

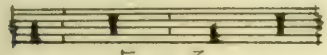

Fig. 3.

The external walls are 12" thick, and they are carried at each floor by the wall beam. Rods are run vertically through this wall at about 2" from the external surface, spaced about 12" on centers. Thus the building is intended to be absolutely monolithic and fireproof.

The Building

The lot is 100'-6" wide and 165' 2 3/4" long (the long dimension is east and west, the short, north and south). Allowing for the water table and base and for the thickness of the wall, I found eleven bays to be required in the length, and seven in the width. This spaces the columns 14.6' lengthwise and 13.7' along the width. The building consists of the basement,

six floors and the attic. The design comprises the following drawings:

Sheet No 1	Basement Plan.
Sheet No 2.	First Floor Plan.
Sheet No 3.	Second Floor Plan.
Sheet No. 4.	South Elevation.
Sheet No 5.	East Elevation and Transverse Section.
Sheet No 6.	Details:- Cols. and footings
Sheet No. 7.	Details:- Floor Slabs and Beams.

Note: The following pages comprise all the calculations made in designing the building. The floor beams and slabs, columns and stair construction, and all the footings. All the calculations, as before stated, are based on the formulas from Taylor and Thompson's work.

Calculations for Floor Slab

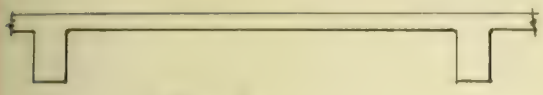

Fig 4

All floor slabs are to be continuous each way and reinforced both ways. This, according to the New York City building reduces the maximum bending moment to $\frac{WL^2}{20}$. As before stated, from the table in Taylor and Thompson's work, the working strength of the steel was taken as 14000# per square inch. Concrete = 550#. $\frac{E_s}{E_c} = 15$.

$E_s = 30,000,000$ $E_c = 2,000,000$. $k = 89$.

The live load capacity was taken as 200# per square foot. Assumed a width of 12" for slab. 250# + 90# (wt. of slab per square foot) = 340# = total live and dead load per square foot. $M_B = \frac{WL^2}{20}$ Span = 14.7' (took longest span, and made same beam do for both spans).

14.7' = 175.2" 340# per square foot = 28.3# per square inch.

Hence $M_B = \frac{WL^2}{20} = \frac{(175.2)^2 \times 28.3}{20} = 43,434$ inch lbs.

$M_B = k\,b\,d^2$

$43,434 = 89 \times 12 \times d^2$

$d = 6.2$" Make slab 7" thick.

The steel area in section 12" wide = $.0073 \times 6.2 \times 12 = 0.543$ sq"

Use 3/4" rods spaced 8" o.c. each way.

Note: For details of floor slab see sheet # 7 of drawings.

Floor Girders.

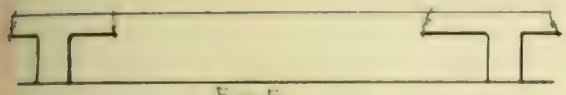

Fig 5.

The live load on floors equals 250# per square foot. The weight of floor slab 7" thick is 90# per square foot, making a total of 340# exclusive of the weight of the girder. Each girder carries 1/4 the load of each floor panel, but 1/4 on each side makes each girder carry 1/2 the load. (wall girders excepted). Assume the girder to be 12" wide. From the tables in Taylor and Thompson, page 154, took a weight of 256# per running foot. The total load on each girder exclusive of its own weight = $(14.6 \times 13.7 \times 340) = 34000$#. Dividing this by the span, 14.6, gives 2329# per foot. 2329 + 256 = 2585 = total load in pounds per running foot. Since the girders are to be continuous the formula for maximum bending moment is $\frac{Wl^2}{10}$.

$$M_B = \frac{(2585 \div 12) \times (14.6 \times 12)^2}{10} = 661{,}170 \text{ inch pounds.}$$

$M_B = kbd^2$. $661{,}170 = 89 \times 12 \times d^2 = 1068 d^2$

$d^2 = 619$. $d = 25$. Allowing 2" below the steel, we have a depth of 27" for the girder.

Area of steel = $.0073 \times 12 \times 27 = 2.13$ □". Use 4-7/8" rods.

[For detail of girders see sheet No 7 of drawings.]

Roof Slab.

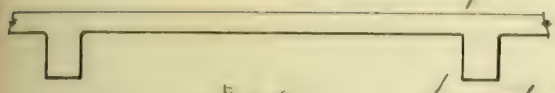

Fig 6

Capacity - 60# per square foot including snow load. Took a section 12" wide. The weight of a 1" slab 12" wide 51# per square foot. Hence the total load is 111#.

$M_B = \frac{(111 \times 12)(14.6 \times 12)^2}{20}$ continuous both ways.

$= \frac{925 \times 30695}{20}$ 14,140 inch pounds.

$M_B = kbd^2 = 89 \times 12 \times d^2$

$14140 \quad 1068 d^2$

$d^2 = 13.2$

$d = 3.7$ Allowing 1.3" for depth below steel, the slab should be 5" thick. The steel area $= .0073 \times 5 \times 12 = .438^{\square"}$ Use a 3/4" rod spaced 12" o.c. each way. (Sheet No 7 for detail).

Roof Girders.

The weight of the roof slab, including the live and dead load = 111# per square foot. Each beam carries 1/2 a roof panel; or - $(111 \times 14.6 \times 13.7) = 11,100$#

Assume a width of 12" for the beam. Took a weight of 204# per foot. $11,100 \div 14.6 = 754$# per foot. $754 + 204 = 958$# or 80# per inch.

$M_B = \frac{80 \times (14.6 \times 12)^2}{10} = 245,560$ inch pounds.

$M_B = kbd^2 \quad 245,560 = 1068 d^2. \quad d = 15.$

Allow beam to be 16" deep. Steel area $= .0073 \times 16 \times 12 = 1.4^{\square"}$ Use 4 - 3/4" rods. (See Sheet No 7 for detail).

Exterior walls

External walls are to be carried on wall girder at each floor. Walls to be 12" thick.

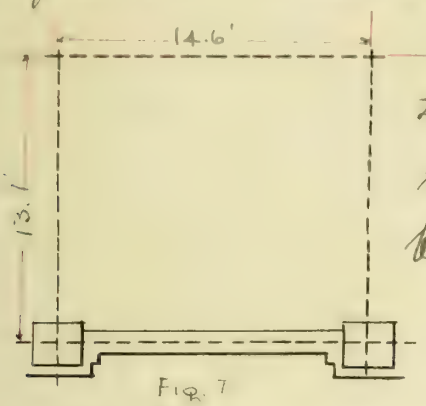

Fig. 7

The columns average 24" in width. Then on the side where spacing between columns is 14.6' the clear span of wall to be carried on the girder is 12.6'. The height from floor to floor is 12.0". But the depth of the girder is 2'-3". So that the area of wall is 9.75 × 12.6 = 147 □'. The weight of concrete 12" thick with reinforcement is 154 #□'. 154 × 147 = 21800# to be carried on girder. Now the wall girder does not carry as much floor load as the interior girders but only ½ as much, making floor slab weight 17000#. Then the total weight on girder is 17000 + 21800 = 38800#.

$\frac{38800}{14.6}$ = 2660# per foot. Allow 372# per foot for weight of girder. Total weight on beam = 2660 + 372 = 2982# per foot.

$M_B = \frac{W l^2}{10} = \frac{(2982 \div 12) \times (14.6 \times 12)^2}{10}$ = 764000 inch pounds.

$1068 d^2 = 764000$

$d = 26.7"$ Allow ~~slab~~ beam to be 28.7" deep. Reinforced with 4 - 7/8" rods.

Girders under silver vault walls

Wall to be 4" thick separated by a 1" air space
Plaster to be all lined inside. The weight
of wall will be 103# per square foot. The steel
will weigh 50# per sq. foot making a total of 153#
153 x 142 = 21700#. The weight of floor slab weight
is $\frac{68000}{2}$ = 34000. Assuming weight of girder
as 332# per foot. We assume a width of 12".

34000 + 21700 = 55700 $\frac{55700}{14.6}$ = 3820.

3820 + 332 = 4152# = total load per foot.

$M_B = \frac{\left(\frac{4152}{12}\right) \times (175)^2}{10} = \frac{346 \times 30600}{10} = 1,300,000$ inch lbs.

$1068 d^2 = 1,300,000$

d = 35" Total depth = 37"

37 x 12 = 444 □" .0073 x 444 = 3.28. Use 4 - 1 1/4" rods

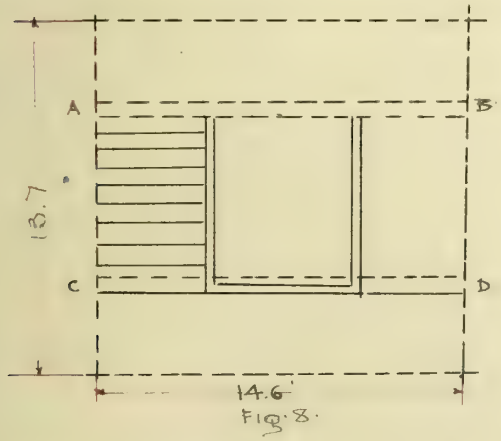

Fig. 8.

Framing for Stairway.

There are two auxiliary girders, one at the spring line, the other at the landing. The span of each = 14.6' Consider each flight of stairs as a floor slab supported at each end on girder CD and A.B. The dimensions of each flight are 3.5' x 10' measured on the slope. = 35 □' Let the live load capacity be 60#. Making a total of 2100#. If

the slab is 7" thick the weight will be 90# per square foot. Making a total load of 3150#. Taking a section 12" wide we find that the live and dead loads # 150# per foot, or 12.5# per inch. $M_B = \frac{12.5 \times (120)^2}{8} = 22500$ inch lbs.

$1068 d^2 = 22500$ $d^2 = 21.2$ $d = 4.6$.

Allowing slab to be 6" thick Area of section = 6" x 12" = 72". .0073 x 72 = .52 square inches of steel.

1 - 3/4" rod spaced 9" each way will be sufficient. Now the steps (7" x 10") are to be molded on top of the slab, giving an extra weight of 1225# This together with the live load of 60# gives the total slightly in excess of the weight figured. But the slab was figured only for reinforcement one way and being reinforced both ways makes it amply strong enough.

Girder A.B. or C.D.

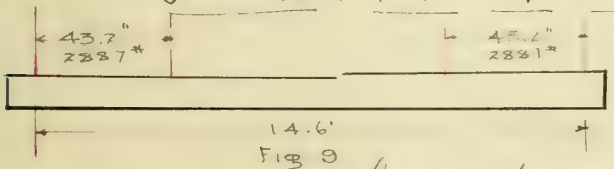

Fig 9

The girder has attached for a distance of 43.2" half the weight of the stair slab or 2887#. 66.7# per inch. It also has half the weight of a floor slab 3 x 14.6 figuring this 7" slab at 103# per foot and the live load at 60# the total is 163# per square foot. 163 x 3 x 14.6 = 8169# 8169 ÷ 2 = 4080# on girder.

280# per foot. Assume a width of 10" for girder. For a depth of 16" it will weigh 110# per foot. 280 + 110 = 450# per foot. The reaction at the support = 6560 + ~~2887~~ + 2887 = 6167#

Max M_B = $6167 \times \frac{17.5}{2}^2 - 2887 \times (\frac{17.5}{2} \times 1.6)$ 3280×43.8

= 206000 inch pounds.

$Kbd^2 = 206000$ $b = 10$ $K = 89$

$206000 = 890 d^2$

$d = 15.25$. Allow beam to be 17" deep. Use 4 - 3/4" rods.

Floor Beam under 4" Partition

Area of partition for long span = 142 □' Weight = 51# per square foot. $51 \times 142 = 7240$ $7240 \div 14.6 =$ 496# From the calculations for floor beams we find that the weight of each beam = 2585# per foot. 2585 + 496 = 3080# per lineal foot.

$M_B = \frac{Wl^2}{10} = \frac{(3080 \div 12) \times (14.6 \times 12)^2}{10} = 789000$ inch pounds.

$1068 d^2 = 789000$

$d = 27.2$

Use a 29" ~~slab~~ beam. $12 \times 29 \times .0073 = 2.54$ □" steel.

use 4 - 1" rods.

Columns

Fig 10.

Live.

The roof slab - 14.6 x 13.7 x 5"

Live load - 60# □' Slab weight = 64# □'

Making a total of 124# □'

Each column has to carry the weight of one slab, except the external wall columns, and the weight of two girders.

The roof beams are 12"x16" and weigh 204# per foot. 14.6 x 204 = 2,980# 13.7 x 204 = 2800#

making a total of 5780.# The weight of floor slab, live and dead equals 13.7 x 14.6 x 124 = 24,800#.

Making a total of 5780 + 24800 = 30580# to be carried by the attic column.

Using formula $p = \frac{C_1 - C}{C(r-1)}$, where $C_1 = 800$, $C = 550$, $r = 15$.

We have $p = \frac{800-550}{550(15-1)} = \frac{250}{7700} = .0324 = 3.24\%$ steel.

30580 ÷ 800 = 38.2 □" A trifle over such column would be plenty large enough; but in reinforced concrete construction, it is economy to keep the same size column as near as possible from basement to roof, so that before deciding on this column or any column above the basement, it will be wise to wait until the latter is determined.

Note: See Sheet #6 for col. details.

6th Floor. The weight of preceding column is approximately 1620# 30580 + 1620 = 32200# to be carried from attic. Floor slab 14.6 x 13.7 x 7" unit linload of 250#/ft + 90#/ft dead weight = 340#/ft
14.6 x 13.7 x 340 = 68000. Weight of one girder = 14.6 x 336 = 4900, the other = 13.7 x 336 = 4100.
32200 + 9000 + 68000 = 109200# Using the same value for C_1 of 800. 109200 ÷ 800 = 136□".
The same percentage of reinforcement determined before of 3.24 will continue to be used. (see col sheet)

5th Floor

Weight of 6th floor column is taken at 1944#. 102 + 1944 = 111,144#.
Weight of floorslab = 68000. Weight of two girders = 9000. 111,144 + 68000 + 9000 = 188,144.
$\frac{188,144}{800}$ = 235□" 16" column would be sufficient. (See col. sheet.)

4th floor 2580 = wt. of 5th floor column.
2580 + 188,144 = 190724#. Floorslab = 68000.
Two beams = 9000. Total = 190724 + 68000 + 9000 = 267,724#. $\frac{267724}{800}$ = 300□" Use 18" col. (see col sheet) would be large enough.

3rd Floor Weight of preceding col. = 2920#.
267724 + 2920 = 270644. 68000 + 9000 + 270644 = 347,644 $\frac{347,644}{800}$ = 430□" A 22" column would do. (see col. sheet)
Weight = 3560#

2nd Floor. 341644 + 3560 = 351,204.

68000 + 9000 + 351204 = 428,204.

$\frac{428204}{800} = 535$. (1 24" Col would be large enough. Wt = 3900. (See col. sheet)

1st Floor 428204 + 3900 = 432,104.

68000 + 9000 + 432,104 = 509,104. $\frac{509,104}{800} = 636$.

Use a 26" column. Wt 4220. (see col. sheet)

Basement 509104 + 4220 = 513,324.

68000 + 9000 + 513,324 = 590324. $\frac{590324}{800}$ = 737 □". Use a 26" column on this floor.

Now having found the size of the basement column which is the largest of all of the columns, we are in a position to go ahead and determine what we want to use for the other columns; Keeping before us the fact that changing the size of column every floor is very pensive, on account of forms. The most economical method would be to continue the same size clear up. On the following sheet I have made a schedule of the columns, giving size on each floor and kind and amount of reinforcement. The percentage of reinforcement from which I determined the basement column reinforcement is 3.24 %.

SCHEDULE OF COLUMNS.

FLOOR	SIZE	HEIGHT	REINFORCEMENT	
Basement	26'x26	14'-6"	4 - 2½" rod/	8 - ½" rod/
First floor	26 x 26	12'-0"	4 - 2½" rod/	8 - ⅞" rod/
Second	24 x 24	12'-0"	4 - 2¼" rod/	8 - ¾" rod/
third	24 x 24	12'-0"	4 - 2¼" rod/	8 - ¾" rod/
fourth	20 x 20	12'-0"	4 - 1¾" rod/	8 - ½" rod/
fifth	20 x 20	12'-0"	4 - 1¾" rod/	8 - ½" rod/
sixth	20 x 20	12'-0"	4 - 1" rod/	
seventh	20 x 20	10'-0"	4 - 1" rod/	

Note: For detail/ of col/. /howing arrangement of reinforcement etc, /ee /heet no: 6 of drowing/.

Loads on footings.

Figuring all of the dead and 50% of the live load.

Lin load on roof slab.	12000
Lin load on 7 floor slabs	350000
Total	= 362000
50%	= 181000.

Dead load = 595184 − 362000 = 233,184#

233,184 + 181000 = 414,184#.

Since Columns are to be 26" in basement and first stories, 24" in second and third, and 20" in all the rest; add 3726# for increase of weight to 414184 making a total weight on footings of 417 910#. Allow a bearing of 5000# per square foot on soil. Hence we would have to spread footing over 84 □' $\sqrt{84}$ = 9.2'

Considering the footing as two cantilever beams reinforced both ways, take a section 12" wide. 9.2' = 110.4".

5000 ÷ 12 = 418# per inch.

$M_B = \frac{Wl^2}{2}$. For we may consider the footing turned upside down, the pressure on the soil being the uniform load, the column acting as the support.

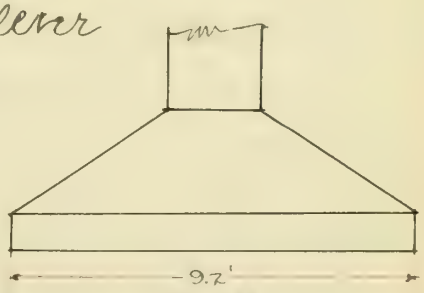

Fig. 11

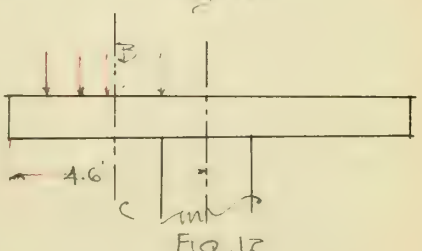

Fig. 12.

The span for each beam would be 4.6'. or 55.2"

$M_B = \frac{418 \times (55.2)^2}{2} = 639000$ inch pounds.

$k = 89$ $b = 12$. $12 \times 89 d^2 = 639000$. $d = 24.5$. to center of steel. Allow 2" below making total depth of 26.5". Taking a bending moment at half the span to find out how to splay the footing in here $M_B = \frac{Wl^2}{8} = \frac{418 \times (55.2)^2}{8} = 158000$.

$1068 d^2 = 158000$

$d = 12.2"$

$.0073 \times 12 \times 26.5 = 2.25$ □" = steel. Use 1 1/4" rods spaced 6" o.c each way.

This footing is detailed on sheet # 6 as well as all the other footings, showing the columns they support.

Wall Column Footings

Each wall column has one half a beam less to support than the interior column, and since this beam is the short one of 13.7 span, weighing 4100#, this column would support 2050# less load at each floor. The wall column has only half a slab to support instead of a whole one. $1/2 \times 68000 = 34000$. $34000 + 2050 = 36050$#. 9000 of the 34000 is dead load. 25000 liv. making a total of 11050 dead and 25000 liv. 50% of 25000 = 12500. $12500 + 11050 = 23550$# which is the amount to be subtracted from each floor load on footing. But the column

has the wall load of 21800" which is all dead. 23550 - 21800 = 1750 at each floor. The roof slab weighs 24800 of which 12800 is dead and 12000 is liv. 1/2 roof beam = 1400. Making a total of (1400+6400) or 7800 dead and 6000 liv. 50% of 6000 = 3000 making a total of 10800 to be subtracted. 7 x 1750 = 12250. 10800 + 12250 = 23050#, to be subtracted from the total load on the interior columns.

417910 - 23050 = 394860

394,860 ÷ 5000 = 79□' Making a base 8.9' square.

Taking a section through the center 1'-0" wide. The span = $\frac{8.9}{2}$ = 4.45' = 53.4"

$M_B = \frac{418 \times (53.4)^2}{2} = 593500$ inch pounds

$1068 d^2 = 593500$

$d = 23.6$. Making total depth 25.6". Use 1¼" rods spaced 6" o.c. each way.

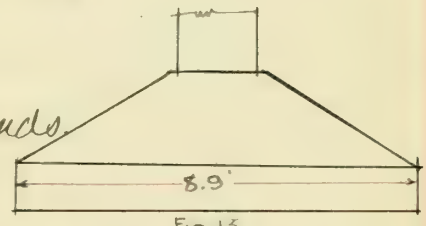

Fig. 15.

Stair well footings

The columns surrounding the stair well both external and internal will have less load than the columns at other points; for while the ~~liv~~ total weight of a typical floor slab = 18000 dead and 50000 liv, the weights of the stair construction = 6300 + 11082 = 17382 dead and 4200 + 5200 = 9400 liv load. Now since ¼ of each floor slab and

its weight down the column to the footings, 50% of the live and all of the dead; therefore each column surrounding the well will have $(18000 \div 4) - (11382 \div 4) = 250^{\#}$ less dead load at each floor and $(50000 \div 4) - (9400 \div 4) = 10150^{\#}$ less live load.

50% of $10150 = 5075$. $5075 + 250 = 5325$

$5325 \times 7 = 37275^{\#}$ = the amount to be subtracted from the total load on interior footings.

$417910 - 37275 = 380635$. Add 7240 at each floor for partition = 50680

$380635 + 50680 = 431315$

$431315 \div 5000 = 86.2$ = area of footing, making the footing 9.3' square.

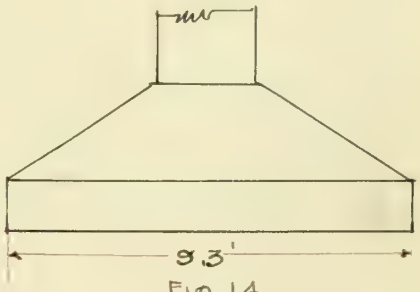

Fig. 14

$9.3 \div 2 = 4.65' = 55.8''$

$M_B = \frac{418 \times (55.8)^2}{2} = 655000$ inch pounds.

$1068 d^2 = 655000$ $d = 24.8''$ Let thickness be 27". Use 1-1¼" rod every 6" each way.

External footing under stair well

Total load on external footings =

394860 $394860 - 37275 = 357,585$.

Add $3620 \times 7 = 25340$ for wall load.

$357585 + 25340 = 382925$.

$382925 \div 5000 = 76.8$. Base will be

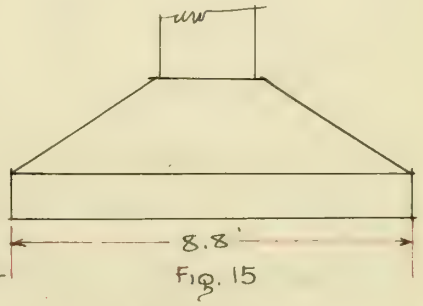

Fig. 15

8.8' square. $\frac{8.8}{2} = 4.4' = 57.8''$ $M_B = \frac{418 \times (57.8)^2}{2} = 582500^{\#}$

$1068 d^2 = 582500$ $d = 23.4$. Let depth = 25.4"

Use 1¼" rods spaced 6" each way.

Footings under interior cols. under partition walls

There will be an increase on each footing of the weight at each floor of the partition between the columns from floor to floor, of 7240# dead load at each floor. $7 \times 7240 = 50680$#

$417910 + 50680 = 468590$ $468590 \div 5000 =$

93.8□' 93.8□' $= 9.7'$ square.

$\frac{9.7}{2} = 4.85' = 58.2''$

$M_B = \frac{418 \times (58.2)^2}{2} = 707500$ inch pound.

$1068 d^2 = 707500$ $d = 25.75$

Allowed for 28" thick.

Use 1 1/4" rods spaced 6" o. c. each way.

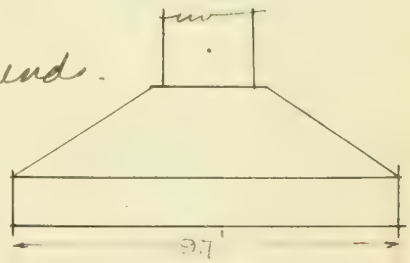

Fig. 16

External Col. Footings under Partition walls.

Increase in weight at each floor will be $\frac{7240}{2} = 3620$

$7 \times 3620 = 25340$#

$394860 + 25340 = 420200$.

This is practically the same load as that on typical interior footing.

External footing will be 9.7' each way. 26.5" thick; using 1 1/4" rods spaced 6" each way.

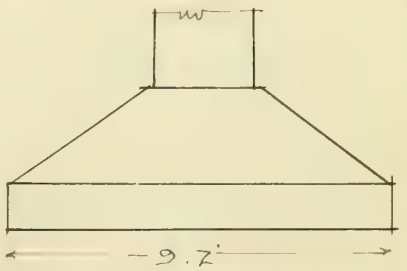

Fig. 17

External Col. footings where selvera vault walls intersect exterior wall.

The load will be increased by

$\frac{21700}{2} = 10850$

$394860 + 10850 = 405710$

$\frac{405710}{5000} = 81. = 9'$ square.

$M_B = \frac{418 \times (54)^2}{2} = 610000.$

$1068 d^2 = 610000$

$d = 24.$ The footing will be 26" deep.

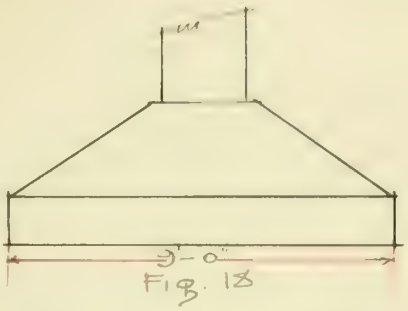

Fig. 18

Use 1"/4" rods spaced 6" each way.

Interior footings under vault walls.

Add 21700 to 417900 = 439.610 #

$\frac{439\,610}{5000} = 87.9$ □' This makes the area 9.4' square.

$\frac{9.4}{2} = 4.7' = 56.4".$

$M_B = \frac{418 \times (56.4)^2}{2} = 665000.$

$1068 d^2 = 665000$

$d = 25$ Giving total depth of 27".

Use 1"/4" rods spaced 6" each way.

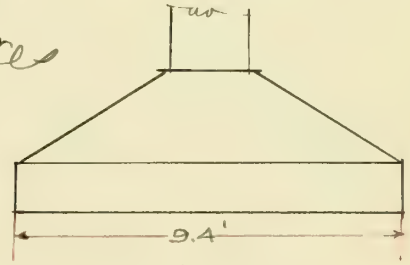

Fig. 19

Footings at intersection of vault walls with Main partition walls.

$417910 + 21700 + 47060 = 486670$

$\frac{486670}{5000} = 97.2$ □' making footing 9.9' square. $\frac{9.9}{2} = 4.95' = 59.4"$

$M_B = \frac{418 \times (59.4)^2}{2} = 736000.$ $1068 d^2 = 736000$

$d = 26.4"$ Set depth for 28.4"

Use 1"/4" rods 6" O.C each way

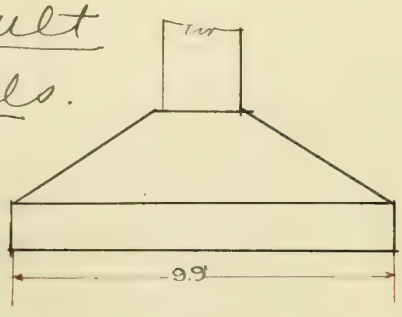

Fig 20

Footings under internal columns at elevators

$417\,910 + (7 \times 3620) = 443\,250$

$\frac{443\,250}{5000} = 88.7$ Area = 9.5' square.

$\frac{9.5}{2} = 4.75'$ or 57"

$M_B = \frac{418 \times (57)^2}{2} = 680000''\#$

$1068d^2 = 680000 \qquad d = 25.5$

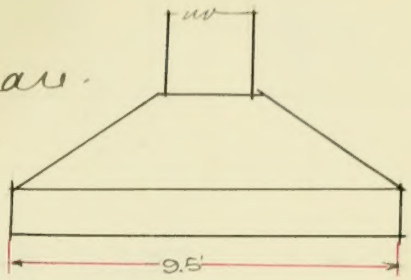

Fig. 21

Total depth = 27.5". Use 1 1/4" steel rods 6" o.c. each way.

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